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New Formulation of the Bearing Capacity of Shallow Foundations Based on the Dynamic Cone Penetration Test (DPT): Analysis of a Database

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ABSTRACT

Among *in-situ* geotechnical tests, the dynamic penetration test (DPT) is less frequently employed as a computational tool for assessing the bearing capacity of shallow foundations. This paper introduces a new direct method, based on the number of blows N_d measured by the DPT, to evaluate the soil bearing capacity of shallow foundations. After describing a database from Algerian coastal soils used for calibration by correlation, the paper focuses on presenting the method and its features. It was found that the bearing capacity factor K_d depends on the foundation slenderness ratio, and can be determined by simple analytical equations or charts for practical purposes. Based on the DPT test, known by its rapidity and the simplicity of its operating procedure, the proposed method compares very well with the well-established PMT-based method. As part of the validation process, a direct comparison of the proposed DPT-based method with the plate loading tests, demonstrated its good predictive capability.

Keywords: Dynamic cone penetration test, Foundation, Plate loading test, Bearing capacity, Correlation.

INTRODUCTION

The dynamic cone penetration test (DPT) is a usual geotechnical *in-situ* test. As illustrated in Figure 1, this test consists of driving a cone connected to a set of rods, using a drop hammer of mass M falling from a fixed height H, and measuring the number of blows N_d required to penetrate the cone to a depth h.

The DPT is the most suitable *in-situ* test for an initial site investigation of large surface areas, aimed at guiding subsequent detailed geotechnical studies. Additionally, this test can be used for qualitative site investigation, such as locating bedrock or detecting underground cavities. Moreover, due to its relatively low cost, it can serve as an effective foundation study tool when the geotechnical investigation budget is limited (Navarette, 2009; Escobar-Valencia, 2015).

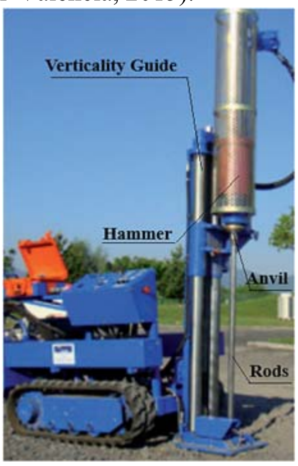

Figure 1. Photo illustrating a typical DPT test device

The DPT is ideal for investigating granular soils or predominantly granular soils. It can also be performed in fine-grained soils above the groundwater table. However, if used in a fine soil layer below the water table, it may lead to inaccurate results. In fact, the driving causes high dynamic point resistance, because the pore water absorbs a significant portion of the driving energy, completely distorting the test interpretation (Sanglerat, 1965; Costet & Sanglerat, 1983; Nuyens,1973). It is therefore recommended to use the DPT for permeable soils or unsaturated low-permeability soils.

From a theoretical perspective, the analysis of the driving response of a slender elastic element, such as a pile, sheet pile, or penetrometer rod, may be conducted either through an energy balance approach, leading to the so-called driving formulae, or via stress wave theory (also known as the theory of stress characteristics). To establish a calculation parameter independent of the geometric characteristics of the DPT apparatus, current geotechnical practice generally avoids reliance on the blow count N_d alone. Instead, a conventional dynamic resistance q_d is determined from driving formulae.

The main advantage of the DPT test lies in the rapidity and the simplicity of its operating procedure (Amar & Washkovski, 1983). This has led to the development of a wide variety of equipment, making the measured number of blows N_d meaningless by itself, since it is closely related to the characteristics of the device used (van Wambecke, 1983). Instead of considering a correction and normalization of this parameter, following an approach similar to that applied to the number of blows N_{spt} obtained from the Standard Penetration Test (SPT), early research works on the DPT test rather defined a conventional dynamic resistance, referred to as R_d, based on pile-driving formulae, which, however, suffer from several limitations. In practice, more than 100 driving formulae (Sieffert, 1987; Withaker, 1976; Bowles, 1997) have been proposed, each differing from the others in terms of underlying assumptions and the parameter values adopted in the governing equations. Gonin (1978; 1999) demonstrated that the widely used "Dutch formula", along with other driving formulae, fails to rigorously satisfy the principle of energy conservation fundamental to pile driving analysis. These formulae, which are based on mechanical quantities assumed to be independent of time, inherently disregard the oscillatory nature of the driving process and the transient character of the forces mobilized during impact. Furthermore, they omit consideration of tip-reflected stress waves, which propagate in opposition to the incident waves induced by successive hammer blows. As a result, the penetration response of the rod is intrinsically controlled by the processes of stress wave transmission/reflection along its length.

In practical applications, a pragmatic approach is used, avoiding the uncertainties related to a theoretical model describing the tip behaviour during the DPT test. It consists of using directly the blow count N_d obtained with a standard DPT device, according to the international standard ISO-22476-2 (ISO, 2005), in an empirical correlation with geotechnical design parameters. As an example, the German standard DIN 1054-100, based on this approach, encompasses some useful correlations of the internal angle of friction φ with N_d for sandy soils, depending on the sand density and its gradation (DIN, 1996; CEN, 2007).

According to many modern geotechnical standards (Eurocode 7, DTR), methods of analysis for the bearing capacity of shallow foundations can be grouped into four main categories; namely: vertical loading tests and physical modeling, adoption of prescribed bearing capacity values, the observational method, and the bearing capacity calculation (CEN, 2004; DTR, 2025).

The first category is based on a direct experimental evaluation of the bearing capacity through vertical loading tests on foundations. The test may be performed at full scale, by loading a prototype foundation, or on a small-scale model, provided that the similarity conditions are satisfied. In practice, the centrifuge modeling test is performed under macro-gravity conditions, which is then interpreted and extrapolated to the prototype foundation scale using similarity laws (Garnier et al., 2007). The second category concerns the prescribed bearing capacity values, which are derived from local experience for a given soil type and structure, in the absence of any particular problem.

The third category consists of an interactive design process where the design is revised during the execution of works, for example, in case of uncertainty regarding the behaviour of the structure, the validity of the calculation method, the hydraulic conditions of the site, or when dealing with a marginally stable site (Allagnat, 2004). The last category of methods involves analytical or numerical design calculations, based on a mechanical model simulating the soil/foundation interaction. The analysis may be analytical or numerical using computational modeling tools, such as the finite element method (FEM) or the finite difference method (FDM).

Methods of calculation of the bearing capacity, hereafter referred to as q_l, and based on the DPT test, may be subdivided into two categories; namely, the direct and indirect methods. The last category allows indirect calculation of q_l based on the dynamic penetration resistance R_d, computed by a driving formula as a function of N_d. These methods, which are the earliest, are based on the following equation relating the allowable stress q_{adm} to R_d and the safety factor F_s:

$$q_{adm} = \frac{q_l}{F_s} = \frac{R_d}{\alpha}. \quad (1)$$

Table 1 summarises the most commonly used methods, where the foundation embedded depth and width are denoted respectively by D and B. In addition to the inherent limitations of the driving formula on which these methods are based, their oversimplified nature should be noted, as they neglect important factors, such as foundation geometry, embedded depth, and the representative resistance R_d used in the preceding equation. As can be noticed, the factor α varies within a considerable margin, from 15 to 50.

Table 1. Summary of the most commonly used direct methods

Author(s)	**F_s**	**α**	**Soil**	**Driving formula**	**Comments**
Sanglerat (1965)	4	20	Granular or purely cohesive	Dutch formula	D/B ≥ 1
Amar and Jezequel (1994)		15-20			
Nuyens (1973) Filliat and Dubus (1981)		20			Water table depth > B
van Wambecke (1982)		50	Sand or clay	Casagrande's formula	
DTU-13.12	3	15-21	Granular		

Relatively recent methods, belonging to this category, allow estimating some mechanical properties, like the internal friction angle of granular soils, by correlation to N_d, which will be input in the usual formula of bearing capacity of Terzaghi (DIN, 1996; CEN, 2007; DTR, 2025).

The second category of methods is the most recent and is based on a direct calculation of q_l as a function of N_d and the characteristics of the DPT device, such as:

$$q_l = f(N_d^e). \quad (2)$$

N_d^e is an equivalent number of blows characterizing the soil involved in bearing capacity below the foundation. The proposed method, presented hereafter, belongs to this category.

OBJECTIVES AND METHODOLOGY OF RESEARCH

This paper introduces a new method for calculating the bearing capacity of shallow foundations based on the Dynamic Penetration Test (DPT). The study begins by outlining the research methodology and describing the database used for analyzing correlations. The proposed method is then presented, followed by internal and external validation procedures to evaluate its predictive capability. Furthermore, a direct comparison is made between the bearing capacity predicted by the new method and that estimated from plate loading tests.

The methodology consists of several stages: first, the correlation between the DPT blow count (N_d) and the limit pressure (P_l) obtained from the pressuremeter test (PMT) is examined across a range of soil types. Next, the PMT-based approach specified in the French standard DTU-13.12 (AFNOR, 1988) is calibrated to derive an equivalent formula for calculating the bearing capacity, denoted as q_l^{DPT}, using an equivalent blow count N_d^e.

According to DTU-13.12, the bearing capacity q_l^{PMT} based on the PMT test is expressed as follows:

$$q_l^{PMT} = K_p.P_{le}^* + q_0 . \quad (3)$$

K_p is called "the PMT bearing capacity factor" and depends on the slenderness ratio D/B, the L/B ratio (L being the foundation length) and the soil nature. $P_{le}*$ is referred to as "equivalent net pressuremeter limit pressure", defined as the arithmetic mean of the "net pressuremeter limit pressure" values within a zone thick of 3B/2 beneath the foundation base. q_0 is the vertical overburden pressure at the foundation base.

Similarly, the bearing capacity q_l^{DPT} may be formulated as:

$$q_l^{DPT} = K_d.N_d^e + q_0. \tag{4}$$

K_d and N_d^e are referred to as "the DPT bearing capacity factor" and "the equivalent number of blows", respectively.

By equalizing the two previous equations, K_d is derived as follows:

$$K_d = \frac{K_p}{\lambda}. \tag{5}$$

λ is a correlation ratio defined as:

$$\lambda = \frac{N_d}{P_l *}. \tag{6}$$

An extensive correlation analysis is carried out through a database of geotechnical investigation studies within the scope of many civil engineering projects. The quality of calibration of the bearing capacity is subsequently assessed by comparing q_l^{DPT} with the bearing capacity q_l^{PMT} predicted from PMT data. Finally, the proposed method undergoes both internal and external validation processes to confirm its reliability and generalizability.

DESCRIPTION OF THE DATABASE

Within the framework of large construction sites located along the Algerian coast, a database was built based on geotechnical investigation reports. The following restrictive rules were strictly applied during the process of selecting the data used in correlations:

- Each couple of measurements (P_l, N_d) must be taken at the same depth,
- Measurements taken in fine soils below the groundwater table were not considered, due to the dissipation of a part of the driving energy through the water during the DPT,
- Each couple of measurements (P_l, N_d) must correspond to the nearest possible cored borehole, to classify the soil at a given depth,
- Only a couple of measurements (P_l, N_d) for which laboratory identification tests were performed and classified according to the USCS system were retained.

In this context, 18 projects across 6 major construction sites in the Algiers and Boumerdès counties were selected to create a database containing 90 pairs of DPT/PMT soundings, with 379 points of correlation across a variety of soils, including sand, clay, and silt. According to the Unified Soil Classification System (USCS), the samples extracted from boreholes revealed 4 soil groups: Highly plastic clays (CH), Low-plasticity clays (CL), Highly plastic silts (MH) and Clayey sands (SC). Based on the margin of P_l*, indicated in Table 1, the samples of fine soils are characterized by a consistency varying from very soft to a stiff soil, while the sandy samples are rather loose to very dense.

The DPT devices used are characterized by a hammer mass M of 63.5 kg that freely falls from a height H of 500 or 750 mm; the number of blows was recorded for a penetration of 200 mm and is hereafter referred to as N_{20}, which categorizes these devices within the DPSH category (Dynamic Penetrometer Super-Heavy) according to the standard DIN-4094 (DIN, 2005).

PRESENTATION AND DISCUSSION OF RESULTS

In Figure 2, a typical curve N_{20}=f(P_l*) is illustrated, where a linear proportionality is noticed. As summarized in Table 2, the values of λ were obtained by linear fitting of the relation N_{20} = f(P_l*), with a coefficient of linear regression approaching 100%, indicating an excellent fit.

The bearing capacity factor K_p, used in Equation 3, to calculate the PMT-based bearing capacity q_l^{PMT}, is given in the French standard DTU-13.12, by simple charts as function of the slenderness ratio D/B, the soil nature and the ratio L/B. The curves K_p=f(D/B) were simply fitted by polynomials of 2nd degree as follows:

$$K_p = a_1 + a_2\frac{D}{B} + a_3\left(\frac{D}{B}\right)^2. \tag{7}$$

Table 2. Summary of the statistics of the correlation N_{20}-P_l*

Soil	Sand		Clay		Silt
USCS Class	SC	SP	CH	CL	MH
Number of couples	35	20	125	139	60
Margin of P_l* (MPa)	0.82-2.85	1.09-7.85	0.24-1.49	0.24-1.60	0.19-1.74
Density/Consistency	Medium dense to very dense		Very soft to stiff		
Mean	6.68	3.52	12.76	8.76	10.07
Margin	5.13- 8.86	2.07- 4.75	7.14 - 18.82	5.87 - 12.5	7.14-13.11
Coeff. of variation (%)	8.9	76.2	20.0	18.6	15.1
$CI_{95\%}$	6.32 - 7.04	3.17 - 3.86	12.30-13.21	8.49 - 9.04	9.68 - 10.46
Value of λ (MPa^{-1})	6.62	3.50	13.0	8.8	9.7
Coeff. of linear regression (%)	86.3	84.7	98.5	90.5	88.4

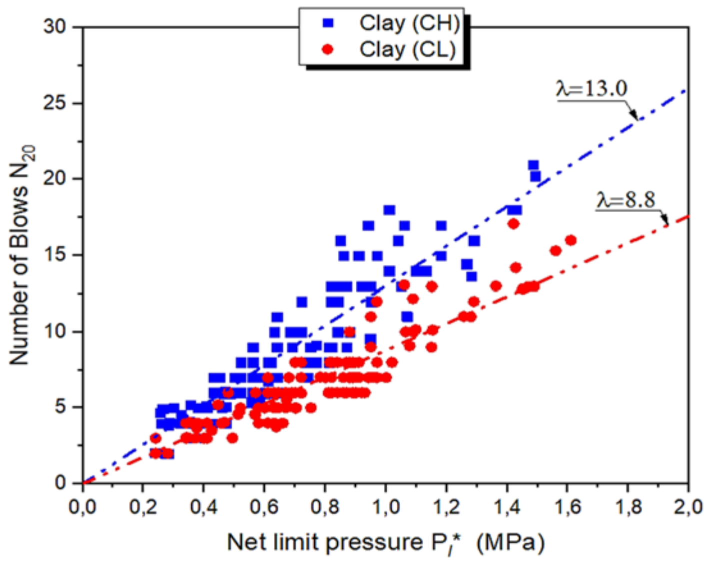

Figure 2. Typical curves N_{20}=f(P_l*) in clays

Based on Equation (5), the DPT-bearing capacity factor, used in Equation 4, may be written as:

$$K_d = \frac{K_p}{\lambda} = b_1 + b_2\frac{D}{B} + b_3\left(\frac{D}{B}\right)^2. \quad (8)$$

Values of b_1, b_2 and b_3 are listed in Table 3, and K_d is shown in Figure 3, where it can be seen that for a surface foundation (D=0), K_d is equal to 0.8/λ, regardless of the soil type and the ratio L/B. Additionally, the chart is limited to the extreme configurations of a shallow foundation, specifically the square foundation (B/L = 1) and a strip foundation (B/L = 0). For practical purposes, the foundation shape effect may be taken into account through the ratio L/B by a simple linear interpolation of the values of K_d between those corresponding to B/L=0 and 1. In this regard, K_d for an isolated foundation (0 ≤ B/L ≤1) may be formulated as:

$$K_d = \left(1 - \frac{B}{L}\right)K_{d0} + \left(\frac{B}{L}\right)K_{d1}\,. \quad (9)$$

Table 3. Summary of the coefficients b_1 through b_3 of the factor K_d

Soil type	**Foundation type**	**b_1**	**b_2**	**b_3**
Clayey Sand (SC)	Square (L/B =1)	0.1190	0.0904	- 0.0278
	Strip (B/L=0)	0.1190	0.0693	- 0.0235
Poorly Graded Sand (SP)	Square (L/B =1)	0.2280	0.1726	- 0.0531
	Strip (B/L=0)	0.2280	0.1323	- 0.0449
High Plasticity Clay (CH)	Square (L/B =1)	0.0615	0.0267	- 0.0074
	Strip (B/L=0)	0.0615	0.0174	- 0.0048
Low Plasticity Clay (CH)	Square (L/B =1)	0.0910	0.0394	- 0.0110
	Strip (B/L=0)	0.0910	0.0257	- 0.0071
High Plasticity Silt (MH)	Square (L/B =1)	0.0800	0.0347	- 0.0096
	Strip (B/L=0)	0.0800	0.0226	- 0.0063

To take into consideration the non-homogeneity of soil properties, the equivalent number of blows N_{de}, as expressed in Equation 4, should be taken as the arithmetic mean of N_{20} within a depth of 3B/2 below the foundation base. It is essential to identify the soil within this zone and verify that it is composed of the same material.

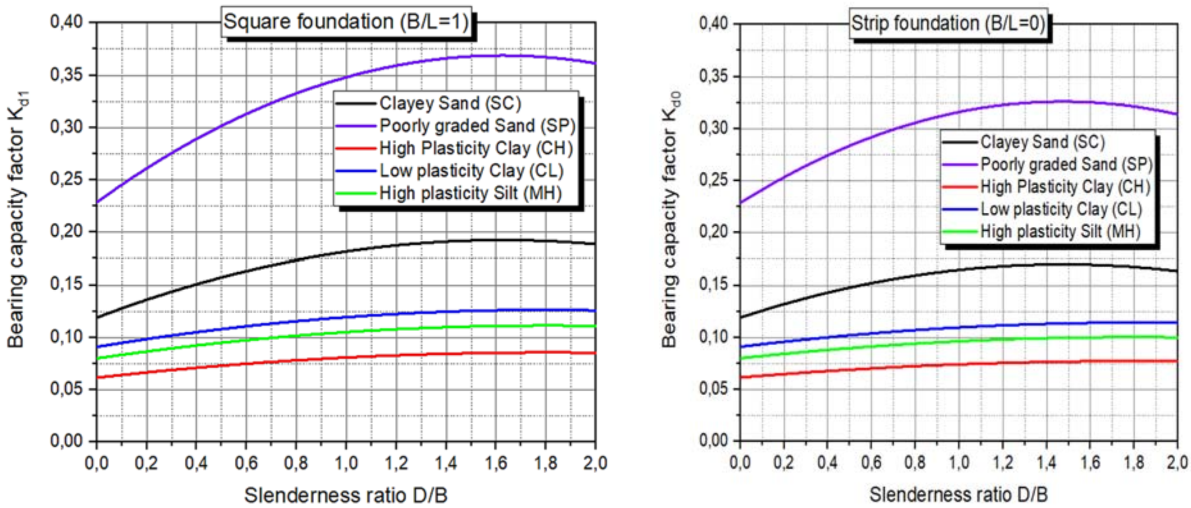

Figure 3. Charts giving K_d as a function of D/B and soil type

Additionally, the distribution of N_{20} with depth within this zone should be filtered by eliminating the peak values, which leads to determining N_{20}^e from a relatively regular profile of N_{20}.

As prescribed by the international standard ISO-22476-2 (ISO, 2005), four possible configurations of the DPT devices may be used. As summarized in Table 4, the first three devices; namely, DPL (Light DPT), DPM (Medium DPT), and DPH (Heavy DPT), should be used to measure the number of blows N_{10} for a penetration of 10 cm. The super-heavy DPT devices DPSH-A and DPSH-B differ in the free fall height and the mass of rods.

The proposed method, developed for the DPSH device (Super-Heavy DPT), which provides the number N_{20} for a 20 cm penetration, can be generalized for N_{10} by using the most commonly used driving formula, called the Dutch formula. The conventional dynamic resistance R_d is a normalized vertical stress designed to be a mechanical parameter independent of the DPT devices, and therefore the same across all DPT devices, as follows:

$$R_d = N_d \frac{M^2 g H}{h S_p (M + m)}. \tag{10}$$

M, m, g, H, h, S_p correspond, respectively, to the hammer mass, the rods mass, the gravitational acceleration, the free-fall height of the hammer, the fixed cone penetration, and the cross-sectional area of the conical tip.

This formula was employed to calculate the N_{20}/N_{10} ratio for various devices. According to Table 4, within a penetration depth z of up to 10 m, the ratio N_{20}/N_{10} varies very slightly with z and ranges from 0.3 to 1.5, depending on the features of the DPT device used. It can be concluded that, regardless of the DPT apparatus employed, N_{20} can be estimated, and the proposed method is applicable for determining the bearing capacity in accordance with Equation (4).

Table 4. Summary of the main features of the standard DPT devices

DPT device	M (kg)	N_d	m (kg)	H (cm)	S_p (cm^2)	h (cm)	N_{20}/N_{10}
DPL	10	N_{10}^L	3z+6	50	10	10	N_{20}/N_{10}^L=0.30
DPM	30	N_{10}^M	6z+18	50	15	10	N_{20}/N_{10}^M=0.70
DPH	50	N_{10}^H	6z+18	50	15	10	N_{20}/N_{10}^H=1.50
DPSH-A DPSH-B	63.5 63.5	N_{20} N_{20}	6z+18 8z+30	50 75	16 20	20 20	N_{20}^A/N_{20}^B=1.00

VALIDATION OF THE PROPOSED METHOD

Internal Validation

The proposed DPT-based method was developed by calibrating the PMT-based method prescribed in the French Standard DTU-13.12 using the coefficient λ. To justify the λ values obtained through correlation and used to define the bearing capacity factor K_d, a direct comparison between q_l^{DPT} and q_l^{PMT} was conducted. This involved simulating three possible foundation geometries: square footing (L/B=1), isolated footing (L/B=2), and strip footing (L/B=10), each embedded at 10 different depths: D/B=0 to 2 with an increment of 0.1, across the sites used to establish the correlation λ. As shown in Figure 4, a strong agreement is observed between the values provided by the PMT-based method, as prescribed in the DTU-13.12 standard, and those predicted by the proposed DPT method. The mean value of the q_l^{DPT}/q_l^{PMT} ratio ranges from 0.97 to 1.15, with a corresponding COV margin of 23% to 27%.

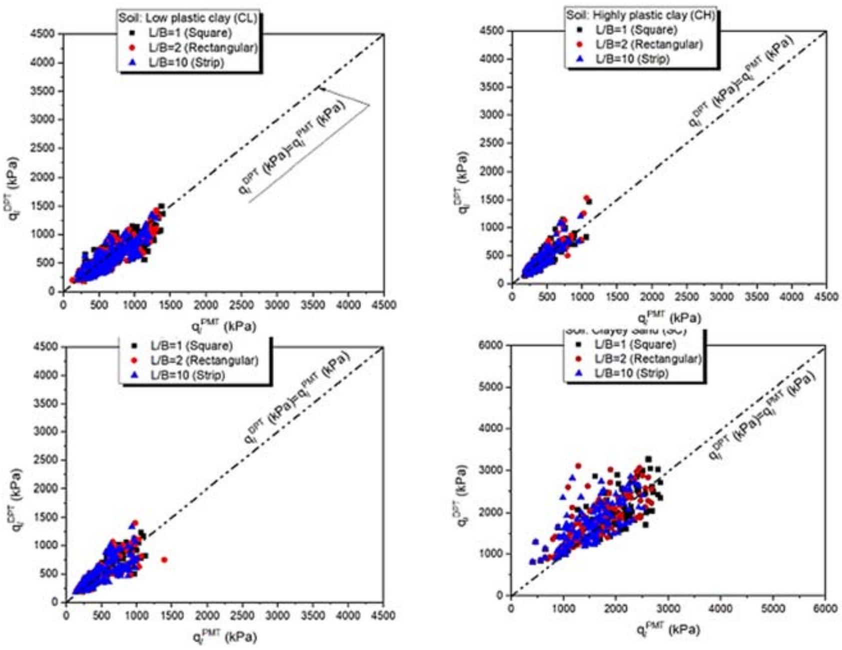

Figure 4. Comparison of q_l(DPT) and q_l(PMT)

External Validation

Analysis of a Plate Load Test (PLT)

Within the framework of a residential building project in the city of Blida, located approximately 50 km south-west of Algiers, two plate load tests (PLT) were performed using a 65 cm-diameter steel plate installed at a depth of 1.8 m below ground level. As illustrated in Figure 5, a sequence of vertical pressure increments was applied, with each load step maintained for a duration of 25 minutes. The soil is mainly composed of a low plasticity sandy clay (CL). The groundwater table is generally encountered at depths ranging from –15.0 m to –20.0 m below ground level.

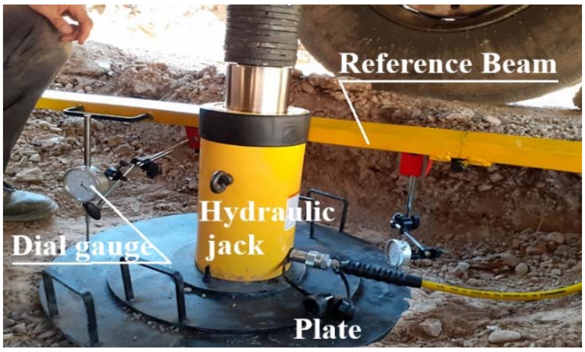

Figure 5. View of the experimental configuration of the PLT test

As shown in Figure 6, the profiles of the blows N_{20} measured within the soundings DPT4 and DPT5, which are very close to the location of the PLT test, exhibit a regular increase with depth below the plate base, from 15 blows to 49 blows. However, the net limit pressure P_l* slightly varies with depth, from 1.6 MPa to 2.5 MPa.

The load-settlement curves of the PLT test are illustrated in Figure 7, where it can be seen that for a maximum pressure of 1025 kPa, the plate settlement reached 5.5% of the plate diameter in the PLT-1 test and 2.4% in the PLT-2 test, indicating a significant difference in soil stiffness at two different locations on the site. Nevertheless, both curves exhibit a strain-hardening behaviour characterized by a regular increase in soil resistance.

Considering these PLT tests as full-scale loading tests of shallow foundations, the bearing capacity may be estimated based on some usual criteria, as summarizedhj in Table 5. However, according to Table 6, a non-negligible discrepancy can be seen between the predicted values.

Statistical analysis of the values of q_l based on the criteria showed relatively high values of COV due to some outlier values. The central tendency of this small-sized population may be characterized by the Hodges-Lehmann estimator, which combines the robustness of the median against outlier values and the efficiency of the mean value (Hodges & Lehmann, 1963).

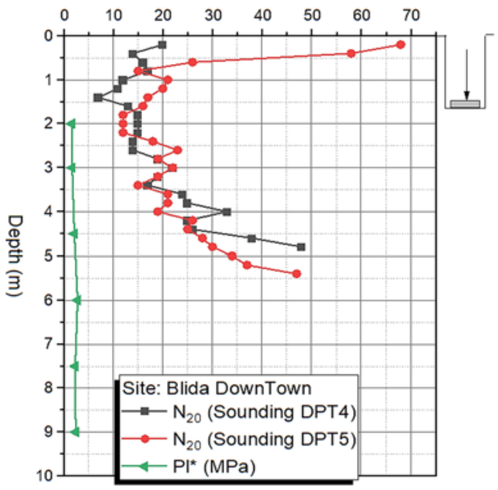

Figure 6. Profiles of the DPT and PMT tests in the Blida city site

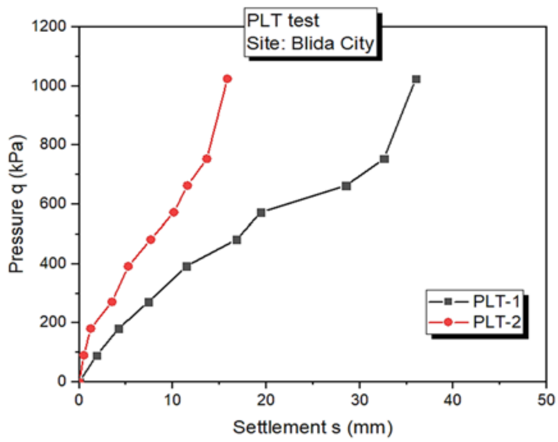

Figure 7. Load-settlement curves of the PLT tests

Table 5. Summary of some usual bearing capacity criteria

Criterion	**Type**	**Formulation**	**Reference**
10% of B	Conventional	q_l= load corresponding to s=0.1B	
Hyperbolic	Theoretical	q=s/(a+bs) q_l= horizontal asymptote of the q-s curve	Duncan and Chang (1970)
Exponential	Theoretical	q=a[1-exp(-bs)] q_l= horizontal asymptote of the q-s curve	van-der-Veen (1953)
Power	Theoretical	$q=as^b$ q_l= load corresponding to s=0.1B	
PARECT	Theoretical	q-s: Parabola-Rectangle function q_l= horizontal asymptote of the q-s curve	Bouafia (2023)
Piecewise function	Theoretical	q-s: Bilinear function q_l= load corresponding to s=0.1B	

In this regard, the most representative value of q_l predicted by PLT-1 is estimated at 1602 kPa, while that predicted by the proposed method is 2531 kPa, representing an excess of 58%. A likely explanation of this discrepancy is that the PLT-1 location may not have been precisely coincident with the DPT/PMT soundings. However, q_l predicted by PLT-2 is estimated at 2735 kPa, whereas the DPT-based method predicted a value of 2307 kPa, with a relative difference of 15.6%.

Table 6. Values of q_l in kPa according to the bearing capacity criteria

Criterion	**Plate Loading Test**	
	PLT-1	**PLT-2**
10% of B	NA	NA
Hyperbolic	1774.0	1388.0
Exponential	1755.2	3660.8
Power	1429.6	2928.0
Parect	1025.0	1025.0
Piecewise function	1833.6	4444.5

NA: Not Applicable.

This comparative study demonstrates a very encouraging agreement between the experimental results of the loading test on a circular shallow foundation and those predicted by the proposed method.

Table 7. Summary of the statistical analysis of q_l according to the bearing capacity criteria

	Plate Loading Test	
	PLT-1	**PLT-2**
Margin (kPa)	1025-1834	1025-4444
Mean (kPa)	1563.0	2689.0
Coefficient of variation COV (%)	21.7	54.3
Median (kPa)	1755.0	2928.0
Hodges-Lehmann estimator (kPa)	1602.0	2735.0
Predicted q_l^{DPT} (kPa)	2531.0	at DPT4: q_l^{DPT} =2322.5 at DPT5: q_l^{DPT} =2291.2 (Mean= 2307.0 COV=1.0%)

Comparative Study with the PMT-based Standard Method

To evaluate the predictive capability of the DPT-based method, a direct comparison was made with the predictions provided by the PMT-based method prescribed by the NF P94-261 standard accompanying the Eurocode 7 (AFNOR, 2013). This method also employs Equation (3), on which the standard method DTU-13.12 is based, but it is fundamentally different, because it relies on the concept of the equivalent embedment depth and a different definition of K_p and P_{le}* (AFNOR, 2013).

This study utilized additional datasets within the framework of 20 projects across 11 sites in the Algiers and Boumerdès regions. The analysis covered the 4 soil classes again: Low-plasticity Clay (CL), Highly plastic Clay (CH), Highly plastic Silt (MH), and Clayey Sand (SC). The population of each soil class was subdivided into 3 data categories (A, B, and C), based on the distance between paired test points (DPT–PMT) and the associated cored boreholes:

- Category A – Closest pairs: distance < 5 m,
- Category B – Intermediate distance: 5–15 m,
- Category C – Widely spaced pairs: > 15 m.

The calculations were performed for square (L/B=1), rectangular (L/B=2), and strip (L/B=6) footings, considering 3 embedment ratios D/B=0.5, 1, and 2, which correspond to 9 geometrical configurations at each (DPT-PMT) sounding.

Figure 8 illustrates a direct comparison of the two methods, and shows that for Category A, where the proximity between DPT and PMT test points was less than 5 m, a very good agreement is noticed, and as summarized in Table 8, the mean ratio q_l^{DPT}/q_l^{PMT} is very close to unity (≈1.0) with very low values of COV lying between 6.6% and 10%.

Regarding Categories B and C, a slight increase in variance was observed as the distance between test locations increased; however, the average ratio remained within the margin of 0.98–1.02, and the COV margins ranged between 7% and 10%.

Table 8. Summary of the statistics of the ratio q_l^{DPT}/q_l^{PMT} (Category A)

Soil	Clay (CL)	Clay (CH)	Silt (MH)	Sand (SC)
Size	324	504	216	189
Margin	0.87-1.32	0.83-1.26	0.86-1.30	0.86-1.33
Mean	0.990	1.01	0.98	1.01
COV (%)	7.5%	9.6%	6.9%	8.3%

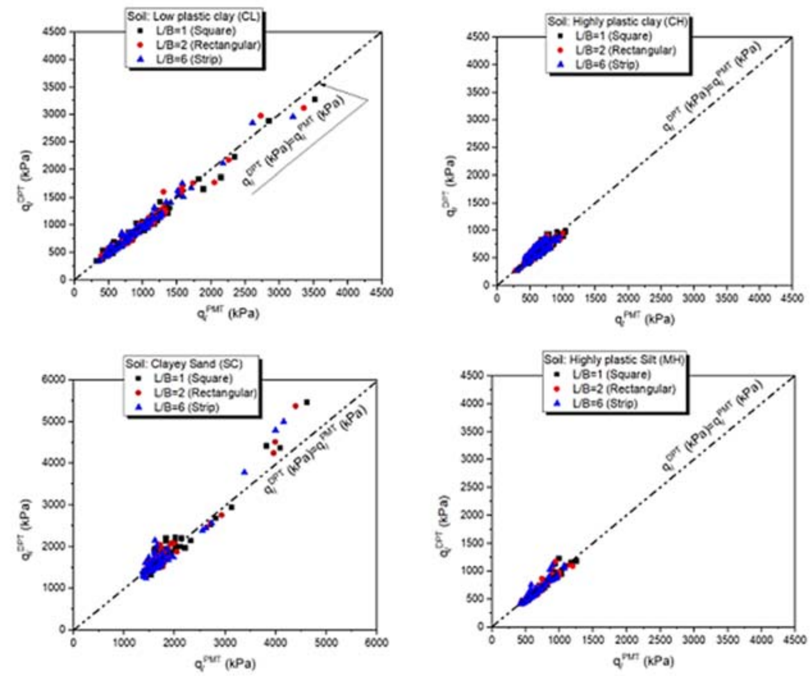

Figure 8. Comparison of the bearing capacities predicted by DPT and PMT (Category A)

These results demonstrate that the bearing capacity values predicted for the four classes of soil by the proposed DPT-based method are practically the same as those given by the PMT method, recently prescribed in the standard NF P94-261. This comparative study thus confirms the robustness and general applicability of the developed method across different soil types.

STEP-BY-STEP PROCEDURE FOR THE DPT-BASED METHOD

Before outlining the methodology for calculating the bearing capacity based on the DPT test, the basic assumptions of this method should be highlighted.

1. The effective bearing zone beneath a footing subjected to a vertical and concentric load is assumed to extend to a depth of 1.5B below the foundation base,
2. The water table is assumed to be below the effective bearing zone,
3. The proposed method is applicable only when the effective bearing zone consists of soil belonging to the same material class. In other words, it is assumed that the soil material is mono-layered. The application of this method to multi-layered soils is a topic of future research. If multiple material classes are present within this zone, the selection of the governing material to be used for calculating the bearing factor K_d is left to the engineer's judgment. It may be possible to assimilate the soil within the effective bearing capacity zone to the material that it most closely resembles.
4. The foundation is a rectangular-shaped footing of dimensions. A circular footing may approximately be treated as a square footing,
5. For safety purposes, in a similar way to the PMT method, any N_{20} value exceeding 1.5 times the smallest value within the effective bearing zone must be bounded at $1.5 \times N_{20}^{min}$, N_{20}^{min} being the smallest N_{20} value.

The step-by-step computational procedure is outlined as follows:

Step 1. For a given geometrical configuration of the foundation to be designed, the soil within the effective bearing zone, thick of 1.5B beneath the foundation base, should be clearly identified and classified according to the USCS system.

Step 2. Compute the equivalent number of blows N_{20}^{e} as the arithmetic mean of N_{20} values within the interval [D, D+1.5B]. All the values should be bounded by 1.5 N_{20}^{min}. In the case where N_{10} were measured by the DPT test, use Table 4 to estimate N_{20} by correlation to N_{10}, depending on the device used during the DPT test.

Step 3. Compute the bearing capacity factor K_d with Equation (9) and Table 3, or alternatively determine it graphically from Figure 3.

Step 4. Compute the bearing capacity according to Equation (4).

CONCLUSIONS AND PERSPECTIVES

As an alternative to some usual DPT-based methods based on the driving formula, the proposed method proposes a direct use of the number of blows to determine the bearing capacity of shallow foundations. A calibration procedure was used to develop this method, based on local correlations.

The application of the proposed method to the case history of PLT tests led to predicting a bearing capacity reasonably close to that obtained experimentally. Moreover, the results of a comparative study demonstrated an excellent agreement between the proposed DPT-based predictions and those obtained by the PMT-based method prescribed in the NF P94-261 standard accompanying the Eurocode 7, with an average ratio close to unity and a coefficient of variation generally below 10%.

The present study shows that the DPT test, when appropriately interpreted, can serve as a simple and effective computational tool for evaluating the bearing capacity of shallow foundations.

As a perspective of this research work, the authors recommend extending the field of applicability of the proposed method to other soil materials.

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Conflict of Interests

The authors declare that there is no conflict of interests regarding this work.

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